89

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DISCUSSION
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451, 469, 544

STRUCTURAL DIVISION

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Current discussion of papers sponsored by the Structural Division is presented as follows:

Number		Page
451	Behavior of Structures Subjected to a Forced Vibration, by Charles T. C. Looney. (June, 1954. Prior discussion: None. Discussion closed)	
	Chang, I. C	1
	Conwell, William A.	4
	Corrections	7
469	Fatigue in Riveted and Bolted Single Lap Joints, by J. W. Carter, K. H. Lenzen, and L. T. Wyly. (August, 1954. Prior discussion: None. Discussion closed)	
	Woodruff, Glenn B	9
	de Vries, Karl	10
	Prince, Harry C.	10
544	Examples of Timber Structure Failures, by Michael N. Salgo. (November, 1954. Prior discussion: None. Discussion closes March 1, 1955)	
	Stern, E. George	13

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Discussion of
BEHAVIOR OF STRUCTURES SUBJECTED
TO A FORCED VIBRATION

by Charles T. C. Looney
(Proc. Sep. 451)

I. C. CHANG,[1] A.M. ASCE.—The elegant simplicity of the procedure presented in this paper should be of interest for practical engineers. Since the nature of this problem is quite similar to that of the stability of rigid frames, an alternative solution is given as follows:

$$y = \frac{2ML^2}{\pi^4 EI} \sum_{n=1,2\cdots} \frac{1}{N^3} \beta_n \operatorname{Sin} \frac{N\pi X}{L} \operatorname{Sin} 2\pi ft$$

where

$$\beta_n = \frac{1}{1 - \left(\frac{f}{f_n}\right)^2}$$

f = The frequency of the periodic end moment

f_n = The natural frequencies of one member simply supported ($n = 1, 2, 3, \text{-----}$)

f_1 = The fundamental frequency of the member

$$\frac{f_n}{f_1} = n^2$$

$$\theta = \frac{dy}{dx} = \frac{2ML}{\pi EI} \sum_n \frac{1}{n^2} \beta_n \cos \frac{N\pi X}{L} \operatorname{Sin} 2\pi ft$$

$$= \frac{M}{EK} \left(\frac{2}{\pi} \sum_n \beta_n \cos \frac{N\pi X}{L}\right) \operatorname{Sin} 2\pi ft$$

$$K = \frac{I}{L}$$

$$\theta_1 = (\theta)_{x=o} = \left(\frac{dy}{dx}\right)_{x=0} = \alpha \frac{M_{ab}}{EI} \operatorname{Sin} 2\pi ft$$

1. Structural Engr. John F. Meissner Engrs., Inc., Chicago, Ill.

where $a = \frac{2}{\pi} \sum_n \beta_n$

$$-\theta_2 = (\theta)_{x=L} = \frac{2ML}{\pi EI} \sum_n \frac{1}{n^2} \beta_n \cos n\pi \quad \text{Sin } 2\pi ft$$

$$= \frac{1}{EK} \left(\frac{2}{\pi} \sum_n \beta_n \cos n\pi\right) \text{Sin } 2\pi ft \cdot M_{ab}$$

$$= b \frac{M_{ab}}{EI} \text{ Sin } 2\pi ft$$

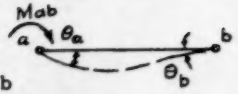


Fig. 1

where $b = \frac{2}{\pi} \sum_n \beta_n \cos n\pi$

Sign Convention

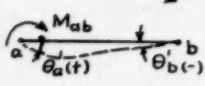
Moment +

θ +

$$\theta_a = \left(\frac{a}{EK_{ab}} \text{Sin } 2\pi ft\right) M_{ab} - \left(\frac{b}{EK_{ab}} \text{Sin } 2\pi ft\right) M_{ba}$$

$$\theta_b = -\left(\frac{b}{EK_{ab}} \text{Sin } 2\pi ft\right) M_{ab} + \left(\frac{a}{EK_{ab}} \text{Sin } 2\pi ft\right) M_{ba}$$

$$M_{ab} = \left(\frac{a}{D} \frac{EK_{ab}}{\text{Sin } 2\pi ft}\right) \theta_a + \left(\frac{b}{D} \frac{EK_{ab}}{\text{Sin } 2\pi ft}\right) \theta_b$$



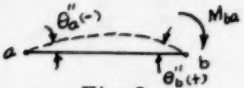


Fig. 2

or

$$M_{ab} = EK_{ab} C_1 \theta_a + EK_{ab} C_2 \theta_b \quad (1)$$

$$M_{ba} = EK_{ab} C_1 \theta_b + EK_{ab} C_2 \theta_a \quad (2)$$

where

$$C_1 = \frac{a \csc (2\pi ft)}{D}$$

$$C_2 = \frac{a \csc (2\pi ft)}{D}$$

$$D = (a^2 - b^2)$$

Eqs. (1) and (2) are of the same form as the slope-deflection equations of the stability of frameworks[2] except C_1, and C_2 here are functions of natural frequency instead of buckling load. It is obvious a system of equations as represented by eq. (1) subject to the condition that

(a) $M_i = 0 \quad (i = 1, 2, 3, \ldots n)$

where n = number of joints

(b) $\theta_{ik} = \theta_{ij}$

Fig. 3

both of which hold true in present case, can always be solved by the method of successive approximation such as the Cross' method of moment distribution or the Grinter's method of balancing end angle changes. For simple structures, the solution can be obtained much quicker by using eq. (1).

2. "Buckling Strength of Metal Structures" by Friedrich Bleich,—McGraw-Hill Book Co., Inc. New York, N. Y. 1952, p. 211.

EXAMPLE

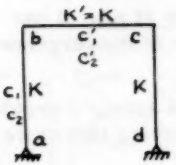


Fig. 4

$$M_{ab} = EK_{ab}(C_1\theta_a + C_2\theta_b) = 0$$

$$\theta_a = -\frac{C_2}{C_1}$$

$$M_{ba} = EK\,(C_1\theta_b + C_2\theta_a) = EK\,(C_1 - \frac{C_2^2}{C_1})\theta_b$$

$$M_{bc} = EK'(C'_1\theta_b + C'_2\theta_c) = EK'(C'_1 - C'_2)\theta_b$$

$$M_{ba} + M_{bc} = 0$$

$$K(C_1 - \frac{C_2^2}{C_1}) + K'(C'_1 - C'_2) = 0 \qquad (3)$$

Or one can write down Eq. (3) directly from its similarity to the buckling problem of hinged symmetrical bent[3]

By substituting values of C_1 and C_2 into Eq. (3) gives:

$$K\,(\frac{a}{D} - \frac{b^2}{aD}) + K'(\frac{a'}{D'} - \frac{b'}{D'}) = 0$$

$$(a - \frac{b^2}{a})\frac{1}{a^2 - b^2} + (a' - b')\frac{1}{a'^2 - b'^2} = 0$$

or

$$a + a' + b' = 0 \qquad (4)$$

Since a, a' , and b' are functions of hatural frequency, these values can be obtained from the curves shown on Fig. (6) given by the author. By using trial and error method Eq. (4) can be solved.

Try $\frac{f}{f_1} = 1.4$ for member BC

$\frac{f}{f_1} = \frac{1.4}{20} = 0.07$ for member AB

$a = -0.07 \quad b = +0.25$

$a' = +0.33 \quad b' = -0.17$

By Eq. (4) Error $E = -0.07 + 0.33 - 0.17 = 0.09$

Therefore $\frac{f}{f_1} = 1.4$ is approximately the correct value.

Since the general slope-deflection equations are well known to structural engineers, this alternative solution may serve for three purposes:

(1) It interprets the significance of the stiffness factor and carry-over factor of the author's procedure[4]

3. "Buckling of Trusses and Rigid Frames," by George Winter, P. T. Hsu, Benjamin Koo, and M. H. Loh, Bulletin No. 36, Cornell Univ. Eng. Experiment Station, Ithaca, N. Y., April, 1948.
4. Statistically Indeterminate Structures by L. C. Maugh, John Wiley and Sons, Inc., New York, N. Y. 1946, p. 132.

(2) It leads to the application of a method of modified stiffness[5] and

(3) It can be used to solve the problem in case the distribution procedure converges very slowly or does not converge at all. It has been pointed out by Dr. E. F. Masur that the range of convergence is limited.[6]

WILLIAM A. CONWELL,[7] M. ASCE.—Coupled with the author's previous work,[8] this paper is a valuable contribution to the engineering literature on vibrations of structures.

In his introduction the author states the intention of describing the "behavior of some relatively simple continuous structures." It should be explained that the method outlined by the author is not limited to the simple structures he has considered but may be extended advantageously to more complicated structures once the feel of the method has been obtained. It is a tribute to the author that he has not only presented the methods, but has actually done the work of solving some of the problems. Instead of concluding his theory with a rather indigestible set of formulae, he has produced pictures such as those of Figure 3. These, while they apply to the simple structures referred to in the introduction, can form a basis for judgment or a first approximation in dealing with more complicated structures and serve as the starting point for further, more accurate computations. While general conclusions from specific data are not always warranted, the examples in this instance are well chosen and pointed to further thinking in the field.

In Table 1 the author shows a range of frequency ratios from 1.13 to 2.11 which he states on Page 9 are grouped in the vicinity of $\frac{f}{f_1} = 1$. From the the practical point of view, it is doubted that a speed of 3600 RPM would be considered in the vicinity of 1800 RPM. It is in the range of hundreds of RPM that the practicing engineer is highly interested. From the point of view from a member with $f_1 = 20$, $f = 1$ may appear to be in the vicinity of $f = 2$. But when one is dealing principally in the first mode of vibration of a structure, where the characteristics of a member having a low fundamental frequency are important, he is not likely to have that point of view. Actually, the influence of a member on the overall natural frequency of vibration of a system is likely to range from $f = 1$ for the case when no moments develop at its end to $f = 2.26$ when no angle changes occur at its ends. The writer prefers to think in terms of this range rather than of frequencies being in the vicinity of the simple span frequency of vibration.

A numerical check of certain of the frequency ratios given by the author in Table 1 was made by an independent method. The values checked were 1.13, 1.18, 1.28 and 1.56. The procedure was an adaptation of the Rayleigh-Ritz method. A mode vibration with its nodes was assumed to have the shape of a static deflection curve with the load acting in the direction of the deflection. Support moments were computed on the basis of the direction of the load determined by the mode of vibration and the following expression for the natural frequencies of a continuous beam of "m" spans was then used:

5. Structural Frameworks, University of North Carolina Press, 1949.
6. "Discussion of Steady-state Forced Vibration of Continuous Frames" Proceedings Separate No. D-136, April 1953, p. 3.
7. Structural Engr., Duquesne Light Co., Pittsburgh, Pa.
8. "Steady State Forced Vibration of Continuous Frames" by Charles T. G. Looney, AM. ASCE, Volume 118-1953, p. 794, Transactions American Society of Civil Engineers.

$$f = \frac{g}{2\pi} \sqrt{\frac{\sum_{m=1}^{n} \frac{w_m^2 \, l_m^5}{24 \times 20 \, E_m \, I_m} \left[4 + 5 \left(k_{Lm} + k_{Rm} \right) \right]}{\sum_{m=1}^{n} \frac{w_m^3 \, l_m^9}{576 \times 1260 \, E_m^2 \, I_m^2} \left[62 + 153 \left(k_{Lm} + k_{Rm} \right) + 186 \, k_{Lm} \, k_{Rm} + 96 \left(k_{Lm}^2 + k_{Rm}^2 \right) \right]}} \qquad (10)$$

where $k_{Lm} = \frac{4 \, M_{Lm}}{w_m \, l_m^2}$ and $k_{RM} = \frac{4 M_{Rm}}{w_m \, l_m^2}$; M_{Lm} and M_{Rm} are bending moments at the left and right ends, respectively, of the mth span; and M is taken as positive when it produces compression on the side of the beam to which the load is applied. An examination of this expression indicates its complete independence of the author's work and the conformance of the results confirms the accuracy of the computations of this paper.

On Page 9 the statement is made "to simplify analysis assume "l" and "I" to be the same for both members." The fixing of "l" and "I" is more than a simplification as the assumption of a different relationship would yield entirely different results. The fact that "l" and "I" are taken as the same for the two members requires, for instance, that the following relationship between the masses of the members is required:

$$m_{12} = \frac{m_{23}}{400.}$$

One might make the observation that this great difference in mass between continuous members of the same structure is unlikely to occur in practice. A further observation might be made that in this particular illustration the "simplification" is not essential as the only change that would be encountered in another assumption would be the revision in the vertical scale of the diagrams for the quantities (a) and (b).

One might observe that, beginning with the expressions (1) and (2) and throughout the paper, the mass of the structure does not enter explicitly into the equations. It, of course, contributes as much to the behavior of the structures as do the "l" and "I" of the members. It actually enters the expressions implicitly through the expression for f_n and hence β_n. The absence of the mass in the expressions should under no circumstances be construed as grounds for disregarding it. The observation might be made that if excitation of a structure is by rotation of an eccentric mass the expressions for the deflections and slopes contain the beam mass explicitly and "I" only implicitly in β_n.

On Page 12 the author states "$\frac{l}{I}$ for both (members) is assumed to be equal." Again it should be noted that this assumed relationship is actually a

condition of the problem and requires that $m_{12} = \frac{m_{23}}{400.} \times \frac{l_{23}^3}{l_{12}^3}$ The behavior of a system depends not only upon the natural frequencies of vibration of its members but upon the way the mass, span and "I" of the members determine their natural frequencies. This may be illustrated by holding "m", "I", and "l" constant for the girder and varying these parameters, one at a time, in the leg to give the required ratio of fundamental frequencies between the girder and the leg of 1:20. The results of such a computation are shown in Table 7.

Note that the frequency of the bent shown in the last line of the table corresponds to that for a uniformly loaded beam with fixed ends and is associated, of course, with small angular rotation at joints 2 and 3.

In computing the values shown in Table 7, the writer found it useful to employ the expression for a symmetrical vibration produced by a periodic moment M sin 2π ft at x = 0 and - M sin 2π ft at x = 1. This expression was obtained by subtracting the author's expression for $\theta_{\widehat{x=0}}$ and $\theta_{\widehat{x=1}}$ giving:

$$\theta_{x=0} - \theta_{x=1} = \left[\frac{1}{2} + \frac{4}{\pi^2}\sum \frac{1}{n^2}\ \gamma_n\right]\frac{Ml}{EI}\sin 2\pi ft \qquad (11)$$

$$n = 1, 3, 5, \ldots$$

Similarly one may obtain the expression for the antisymmetrical case in which periodic moments M sin 2π ft are applied at each end of the beam giving:

$$\theta_{x=0} = -\theta_{x=1} = \left[\frac{1}{6} + \frac{4}{\pi^2}\sum \frac{1}{n^2}\ \gamma_n\right]\frac{Ml}{EI}\sin 2\pi ft \qquad (12)$$

$$n = 2, 4, 6, \ldots$$

Again the writer compliments the author for this work in the field of great interest to civil engineers who are charged with the responsibility of supporting costly and delicate machinery safely and with definite limitations on the amount of vibration.

TABLE 7

Ratio Leg Parameters / Girder Parameters			Natural Frequency For Bent	Angular Rotation Joints 2 & 3
m	l	I		
$\frac{1}{400}$	1	1	1.39	0.333
1	$\frac{1}{\sqrt{20}}$	1	1.83	0.0749
1	1	400	2.26	0.0001

Comparison of Natural Frequencies of Vibration For Bent

In Which $\frac{f_1 \text{ (girder)}}{f_1 \text{ (legs)}} = \frac{1}{20}$

CORRECTIONS

Page 2. The footnote should be: "Expressions for coefficients (a) and (b) on page 14.

Page 5. In the first line, "for" should be capitalized; and in the middle of the page, the equation "$M_{32} = M_{34} = 0$" should read "$M_{32} + M_{34} = 0$."

Page 14. The expression "$f_n^2 = n^2 f_1$" should be "$f_n = n^2 f_1$."

Page 14. In the middle of the page, the expression, "Therefore the values of (a) and (b) are," should be augmented to: "Therefore the values of (a) and (b) are those given in brackets in the following expression:"

Discussion of
"FATIGUE IN RIVETED AND BOLTED
SINGLE LAP JOINTS

by J. W. Carter, K. H. Lenzen, and L. T. Wyly
(Proc. Sep. 469)

GLENN B. WOODRUFF,[1] M. ASCE.—The use of high tensile bolts as a substitute for field rivets is increasing rapidly. This change in practice is mainly for economic reasons but, in general, has been justified by the large program of investigation by the Research Council on Riveted and Bolted Structural Joints of which the studies reported by the authors are typical. These investigations coupled with field experience have proven that in most applications the high tensile bolts are at least the equivalent of rivets. In some applications, especially where vibrations are concerned, the bolts are superior. There are, however, some applications where, the writer believes, it has not been demonstrated that high tensile bolts are satisfactory.

The authors' hypothesis fails to explain why the number of failures have been so few or why they have been confined to hangers since, probably, the average unit stresses in the adjacent diagonals are in the same general order. A crack at the edge of the hole resulting from punching or drilling is a possible explanation; it fails to explain that the failures are confined to the hangers. The writer believes the most obvious explanation is that, for some reason, the stress distribution across the hanger is not uniform.

It has been assumed that, in a material with the ductility of structural steel, these excessive stresses at the edges of a rivet hole would be relieved by plastic flow. However, if a small notch exists at the edge of a hole, the stress concentration will be increased. It is common experience that when a crack starts it continues to lengthen until failure occurs.

The writer does not question the authors' conclusions that a high clamping force reduces the slip and the stress concentration around the hole. However, there are many conditions when comparatively large slips are desirable. In a long riveted splice the design is based on the assumption of equal distribution of shears among all the rivets. It is well known that this never fully occurs and even, as a design assumption, is justified only by the fact that a stress readjustment results from a very considerable slip of the end rivets. Again, measurements of secondary stresses in budges have indicated that such stresses are below those calculated. The most plausible explanation appears that of a stress readjustment as a result of slip. For these reasons, the writer would hesitate to use high-tensile bolts in long span bridges.

The writer applauds the authors' warning that stress raisers should be avoided in all structures. In addition, the writer suggests that in no case should a structural steel in which the yield point exceeds 60 percent of the ultimate strength be used.

1. Cons. Engr., San Francisco, Calif.

KARL de VRIES,[2] M. ASCE.—The authors, referring to reports on fatigue failures in floorbeam connections of railway bridges, are discussing the more general case of the single lap joint. They present a hypothesis relating the fatigue strength of a structural member to the magnitude of tensile stress and strain concentrations at the rivet or bolt hole, and propose as remedy for fatigue failure the use of high strength bolts instead of rivets.

The authors approach the problem by considering first a hole in a plate subjected to tension, and then a hole to which the load is delivered in bearing. The stress concentration factors for these two cases are given in the text as 3 and 6, respectively. They seem to be rather high, particularly to a structural engineer who is used to refer them to the net sections of the plates. If they are referred to the net sections, the concentration factors will be 2.5 and 3.7, respectively, if computed for Figs. 4 and 6a.

The stress concentration factor for tension plus bearing on the contact face of a single lap joint is given as 26, see Fig. 12; but the rivet shank in that plastic model does not fill the hole, and rivet heads are not used; a similar structural joint would fail if subjected to the smallest loads. Actually, a rivet shank will fill the hole, the rivet heads will be stress relieving since they supply clamping forces and, besides, the rivet heads will be stress participating.

The photoelastic studies of the plastic models shown in Figs. 13, 14 and 15 are to indicate the effect of the clamping force of a bolt, without simulating any other forces or their resulting stresses; the force is applied over an area which is equivalent to that of the bolt head. Actually, a high tensile bolt will have a thin washer which extends considerably beyond the bolt head area. The clamping force thus is distributed over a larger area and will not cause the high concentrated surface stress at the edge of the washer which is given for the models. Neither should it be assumed that a high tensile bolt supplies a clamping force and that a rivet does not; only the magnitude of their clamping forces differs.

Fig. 9 is given by the authors to indicate the overload on an end rivet of a joint which is provided with five rivets. It should be noted that only two rivets will be required for that joint to have a balanced design, and that the portion of the total load which was recorded for that end rivet will not necessarily mean an overload for the rivet.

The authors have made a valuable contribution to the knowledge of the fatigue behavior of rivets and bolts in simple lap joints. To prove a hypothesis for that case, it seems, it is necessary to simulate the actual conditions more closely. The remedy probably is the use of high tensile bolts instead of rivets. Nevertheless, it should be said that rivets are not as poor as presented in the paper.

HARRY C. PRINCE,[3] A.M. ASCE.—The documented results presented in this paper amplify and verify many studies on the use of fasteners for structural steel construction. There are structures in service which have had similar failures as exampled in railroad floorbeam hangers, but occurring in diagonals of trusses.

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2. Engr., Bethlehem Steel Co., Fabricated Steel Construction, Bethlehem, Pa.
3. Designer, American Bridge Div., U. S. Steel Corp., Chicago, Ill.

These studies raise questions which are presented as follows:

(1) Since there have been relatively few failures in floorbeam hangers as stated by the authors, are the use of riveted details the cause of these failures or are the details and members not designed with proper stress distribution in the members, details or frame?

(2) Are there typical failures only in members where the thickness of member material is appreciably thinner than the gusset material? Does gusset material fail where it is lighter than the member material? Is there a ratio of member area to gusset area that is of importance where fatigue is present?

(3) If, as the authors have intimated, drifting can cause minute fractures at holes in connections, should members subjected to fatigue stresses be forcibly bent to oppose secondary bending stresses?

(4) The indication of plate failure along the edge of the bolt head or washer of high tensile bolts raises the question of load distribution within a large group of such bolts. In riveted joints a stress distribution is possible with slippage of the joint material in the end rivets of the group. High tensile bolts could conceivably restrict such readjustment.

(5) What is the effect of high tensile bolts when fatigue and secondary stresses within large joints are present? May they be safely used in such cases?

The writer believes that more extensive research on the effect of high tensile bolts in large joints common in long span and movable bridges is needed.

The writer agrees with the authors "that further studies of the effect of clamping upon local stress distribution are desirable," especially in applying high tensile bolts to large joints.

Discussion of
EXAMPLES OF TIMBER STRUCTURE FAILURES

by Michael N. Salgo
(Proc. Sep. 544)

E. GEORGE STERN,[1] M. ASCE.—In his paper, the author refers to timber structures assembled with split-ring connectors and bolts, without going into detail as to the type of split-ring connector employed for the structures under discussion.

Three types of split-ring connectors have been used in this country. The original-standard split-ring connector made of flat steel strips is still sold commercially. In 1939, the split-ring connector with inside bevels was introduced and a few years later the split-ring connector with inside and outside bevels was placed on the market, replacing his immediate predecessor.

Because of the apparent inherent advantages brought about by the use of bevelled split rings in comparison with flat split rings,[2] it is of interest whether flat or bevelled split-ring connectors were used for the structures which showed joint failures. The bevelled split ring tightly fits into its precut groove. Consequently, both initial slip and joint deformation during load application can be minimized. This results in reduced truss deflection and smaller superimposed secondary stresses than encountered with use of flat-sided split rings.

In light of the importance of decreasing secondary stresses in structural assemblies, information on field experiences with flat-sided versus bevelled split-ring connectors would be elucidating.

1. Research Prof., Virginia Polytechnic Inst., Blacksburg, Va.
2. See, Stern, E. G., A study of lumber and plywood joints with metal split-ring connectors, The Pennsylvania State College Engineering Experiment Station Bulletin No. 53, 1940.

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